Anti-subsidence Measure Using King-post Cables against Subsidence in A PC Box Girder with Central Hinge

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Abstract: Four dyvidag bridges with hinge on the Hanshin Expressway Network were found subsiding in the central hinge region. Displacement was most significant in the Kireuriwari Viaduct of the Osaka Matsubara Route which exhibited a subsidence of more than 30 cm from the initial level. Anti-subsidence measure was investigated as a part of the major repair work on this route in fiscal 2003. Through comparative studies between a composite truss, external cables installed inside the box girder and other proposals, a king post cable structure was chosen because of its advantages in terms of local and structural conditions of the bridge. This method was to install new struts to the underside of the main girder and lay external cables eccentrically. This report covers detailed design of the king post cable reinforcement technique.

Keywords: continuous three-span PC rigid frame bridge with hinge, sag, king post cables, ASR, major repair work, soundness evaluation, maintenance plan

1. Introduction

The Kireuriwari Viaduct is a continuous three-span prestressed concrete (PC) rigid frame bridge with hinge completed in 1979. Photo.1 shows the whole view of this 154 m long bridge. This type of structure was used in many bridges built in 1960s to 1980s, being considered favorable for long bridges with not so high piers. In such structures with hinge dead load bending moment in a

Photo.1 View of the entire Kireuriwari Viaduct

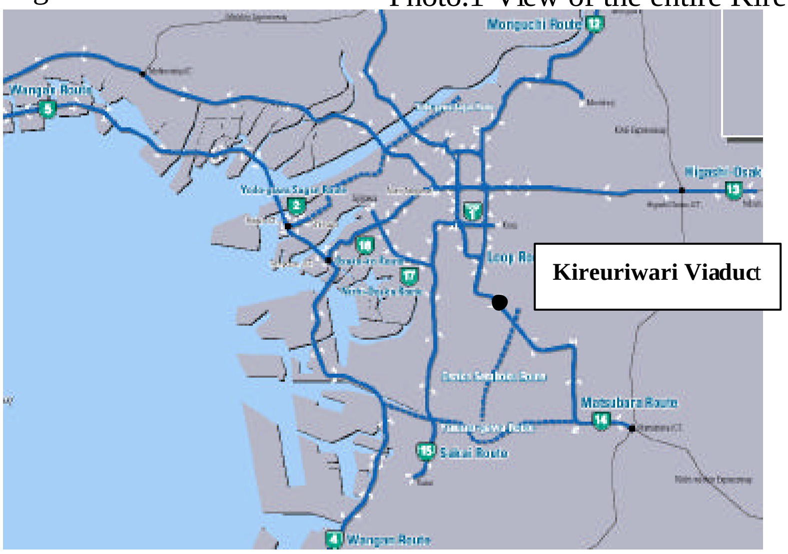

Fig.1 Hanshin expressway network

completed structural system, horizontal seismic force is distributed to each pier, and there is no redundant force due to temperature change and drying shrinkage which usually occurs in ordinary continuous rigid frame bridge. However, many of them have been changed into continuous girders since hinge shoes required frequent maintenance and, in some of them, sagging or angle-bending was beyond design levels, causing problems with driving comfort and water drainage from bridge decks.

The Kireuriwari Viaduct experienced the same problems, with a sag in the central hinge region found in 1985, after five years from the start of service in March 1980. Although overlay of 70 mm was applied in 1986 and again in 1993 in order to sustain serviceability, continued observation and examination suggested a possibility of further decline in its structural performance. The amount of sag already reached a considerable degree and was expected to further increase. Dynamic characteristics of the concrete indicated deterioration due to alkali-silica reaction (ASR). Since combined effects of creep in the concrete, ASR and other factors were suggested, it was decided to restore serviceability and prevent or even correct the sag using the king post cable reinforcement technique. This report covers the detailed design of this reinforcement.

2. Current condition of the existing bridge

2.1 General

Table.1 and Fig. 2 show specifications and general views of the bridge which passes over the Uriwari intersection in Hirano-ku, Osaka. Since this site was a major intersection of two national roads (#309 and #479) with a daily traffic inflow of about 76,000 vehicles, the construction must be carried out under many restrictions including assured visibility for vehicles on the streets, compliance with clearance limits and limited space available for the work.

Table.1 Specifications of the bridge

Structure	continuous 3-span prestressed concrete rigid frame twin main girder bridge with hinge
Bridge length	154.0 m (span lengths: 44.450 + 65.000 + 44.450 m)
Width	0.400 + 8.700 + 0.800 + 8.700 + 0.400 = 19.000 m
Live load	B live load (TL-20 at construction)
Gradient	incline: 1.167%; crossfall: 2.0% (upward gradient)
Bearings	8 roller bearings, 162 tf (overall)
Central hinged shoes	4 Gelenk shoes + 2 horizontal rubber bearings (overall)
Concrete strength	superstructure: 40 N/mm^2; substructure: 24 N/mm^2
Prestressing steel	internal cable: SBPR 930/1180 (dia. 26 mm; existing) external cable: SWPR7BL 19S15.2

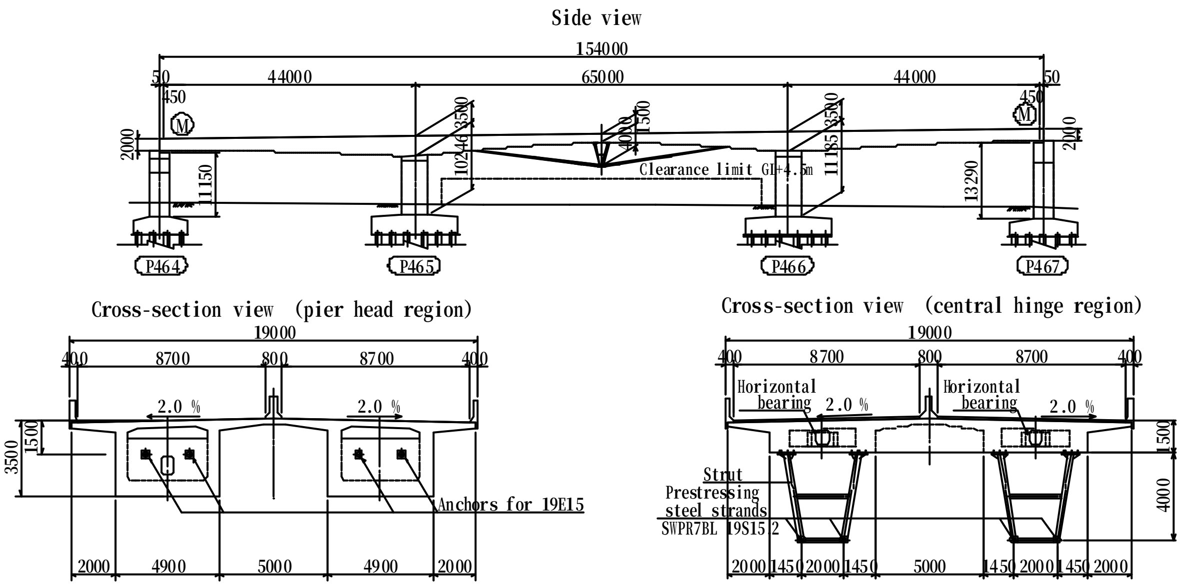

Fig. 2 General structural views of the Kireuriwari Viaduct

2.2 Changes in the bridge from the completion

(1) Sag in the main girder

Figure.3 shows the amounts of sag in the main girder. The bridge began to bend sharply at around 10 m from the center of the span, with minor displacement in the side spans. The overlays for serviceability recovery in 1986 and 1993 caused deflections of 4 mm and 7

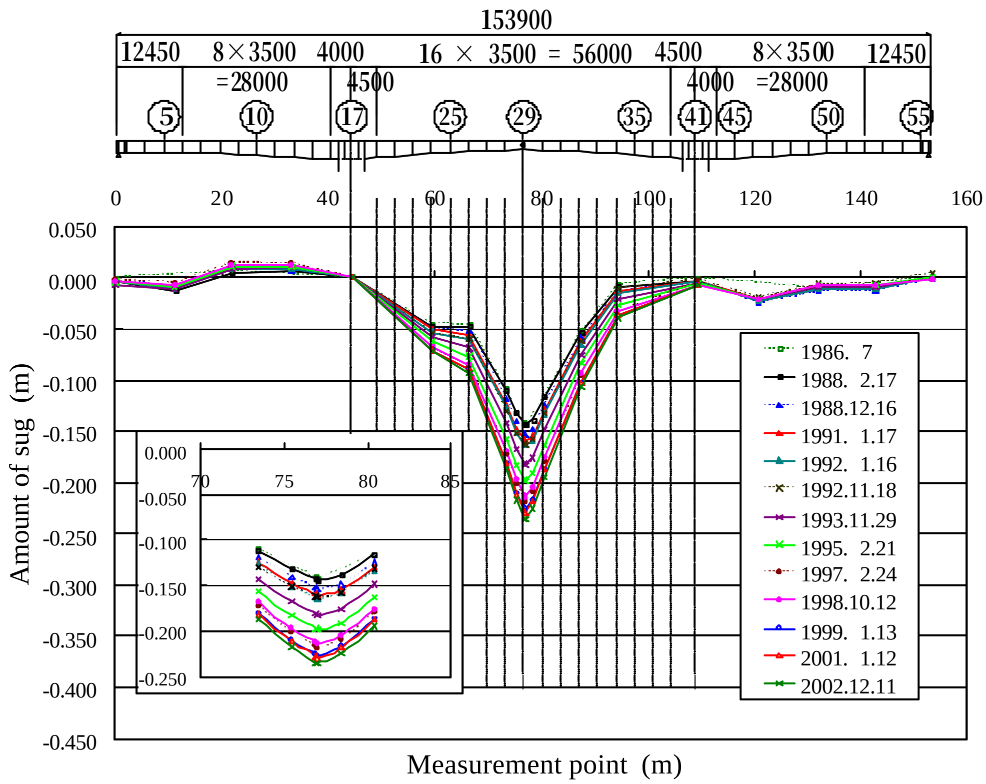

Fig. 3 Measurement results: cumulative amounts of sag (the inbound lane)

mm, respectively, in the central hinge region. None of the piers exhibited major subsidence. Cumulative amount of sag in the central hinge region measured in December 2002 was 236 mm from the design longitudinal alignment. On the other hand, according to the records, this bridge was given an additional elevation of 85.4 mm at construction. Therefore, the actual cumulative amount of sag reached 321 mm. Although there was some influence of temperature variations between measurements, recent data revealed that the rate of sagging was increasing by about 2 mm to 9 mm each year.

(2) Concrete properties

Table.2 shows the results of investigation on concrete properties. Although the design characteristic strength was 40 N/mm^2, actual compressive strength ranged from 28.0 N/mm^2 to 45.5 N/mm^2, suggesting lack of strength partially. Static modulus of elasticity was 60% to 80% of a design level of 3.5×10^4 N/mm^2. Chemical tests and petrographic examinations revealed potential deleteriousness, indicating ASR as one of the possible causes of sagging.

Table.2 Investigation results: properties of the concrete

Year of test	Sampling location	Compressive strength (N/mm^2)	Static modulus of elasticity ($\times10^4$ N/mm^2)	Expansion (1×10^{-6})
1987	Diaphragm	36.4--42.2	2.07--2.54	Mortar-bar: 327 (26W) Chemical: potentially deleterious
1988	Diaphragm	28.0--44.8	1.88--2.72	325--414
1991	Diaphragm	--	--	153--295
1992	Diaphragm	38.4--45.5	2.45--2.55	--
2000	Web	29.7--30.6	2.13--2.34	JCI-DD2: 270 (13W) Canadian Std: 830 (28-day)

(3) Cracking

Cracking of the major part of the bridge was slight. However, alligator cracking occurred in the pier head regions, cross beams and other massive regions. Cracks in the slabs were found along the prestressing steel. In 1988, diagonal cracks were found in the central span at around one fourth the span length from each end. Cracking was significant in the outer webs which were exposed to environmental variations. Investigations were carried out to determine cracking behavior and influence of torsion, resulting in a reinforcement using the steel plate bonding technique in 1993.

3. Detailed design of the king post cable reinforcement

3.1 General

Figure.4 shows the overview of the detailed reinforcement design which consists of the following major items.

(1) Detailed design of the king post cable reinforcement
(2) Concrete property investigation for the main girder
(3) Measurements during prestressing, and existing bridge loading tests
(4) Soundness evaluation, and future deformation estimation
(5) Medium-range monitoring plans

As steps (2) to (5) are still under examination, this report mainly covers step (1).

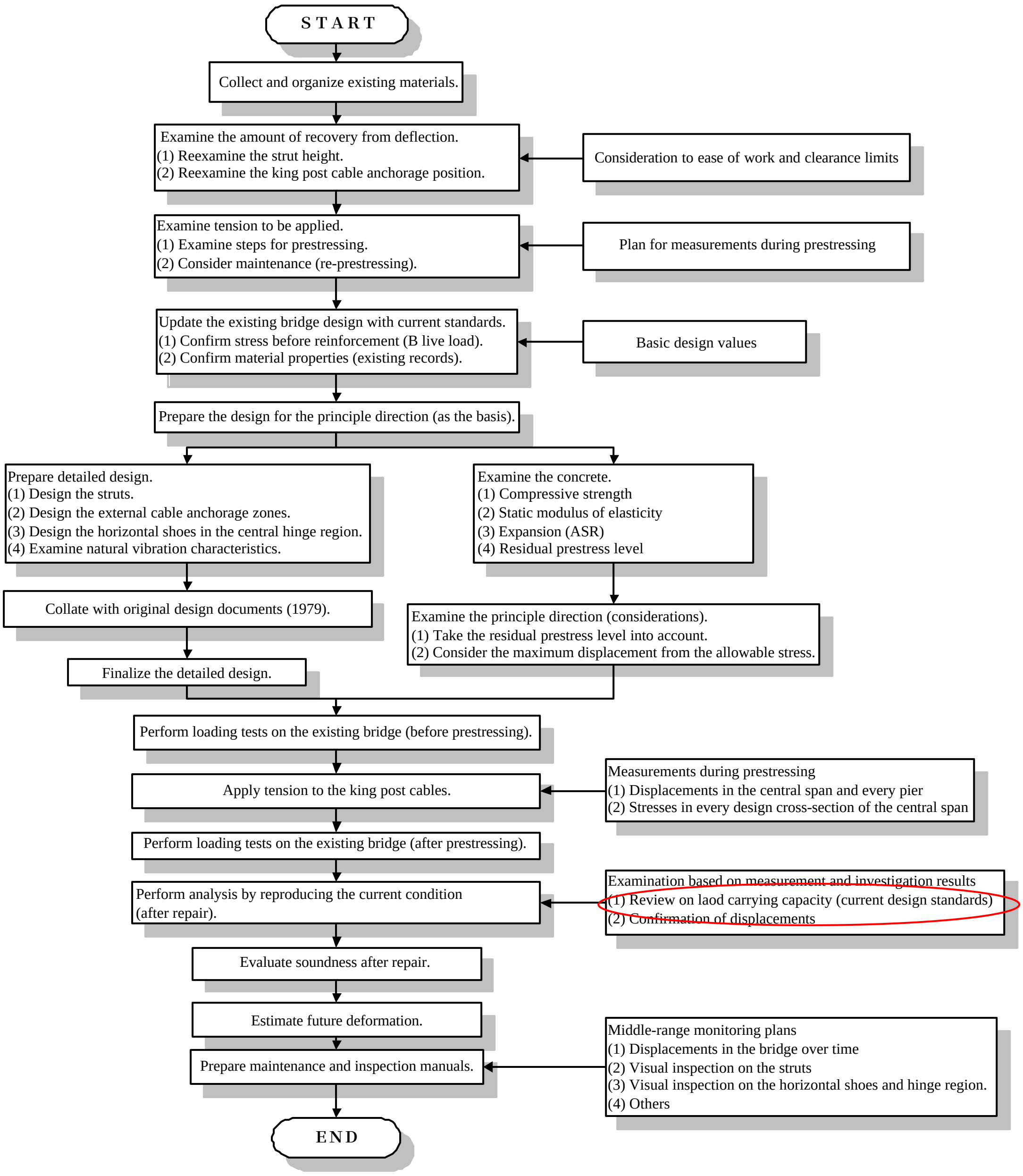

Fig. 4 Flowchart

3.2 Reinforcement in the principal direction

In order to design a continuous rigid frame box girder having the external cables laid with large eccentricities, the elastic plane analysis was used with the king post cables taken into account as members resisting the axial force only. The positions of the king post cables were determined to allow the largest possible eccentricities for optimum recovering effects. Anchoring position was set to 1.5 m from the upper slab where the existing transverse prestressing cables in the pier head region were not interfered with. Struts height were 4.0 m and was placed under the main girder, assuring good visibility for vehicles on the streets and complying with clearance limits.

Tension to be applied to the king post cables was set to initial tensile force σ_{pi} = 1200 N/mm^2 (pi = 3,162 kN/cable), taking stress variations due to live load and temperature changes into account.

Modulus of elasticity of the main girder concrete was obtained from trial mixing results reported during construction. The recorded value was about 2.5×10^4 N/mm^2, which was almost 70% of the original design value (3.5×10^4 N/mm^2). However, since the mix was actually used in the existing bridge, the figure was used as a design value for the current reinforcement.

Figure. 5 shows combined stress under dead load. The current reinforcement was found favorable in terms of stress, with compressive stress due to bending in the central span decreased to around 2.0 to 6.0 N/mm^2. A sag correction of 47 mm was estimated in the central hinge region.

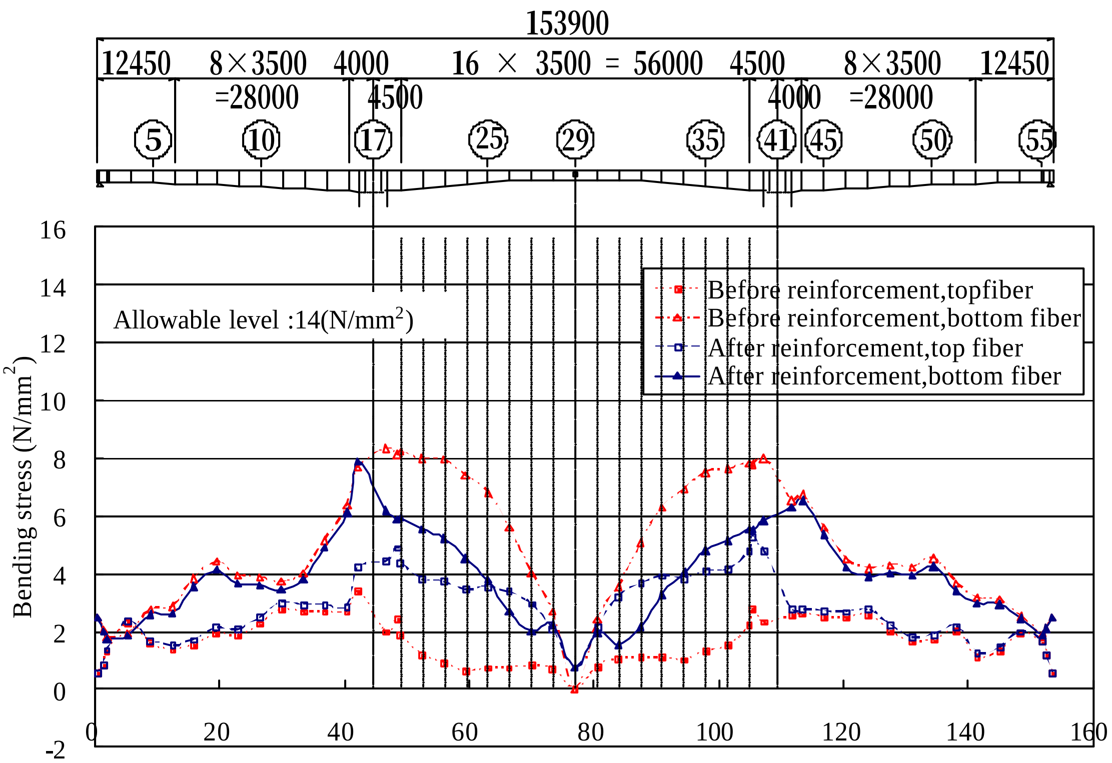

Fig. 5 Combined stress under dead load

3.3 Details

(1) Struts

Struts and saddles were designed to have a simple and light-weight structure, using simplex steel pipes for the struts and duplex steel pipes filled with shrinkage-compensating mortar for the saddles[5)]. Struts had ribs at their roots, with the pipes filled with shrinkage-compensating mortar for the height of the ribs in order to prevent stress from concentrating at the tips of the ribs.

Members were designed using the three dimensional (3D) frame analysis, and local stress was examined using the 3D FEM analysis. Since the struts were crossing over the hinge, relative displacements between the two sides were measured at site and included in the design.

Figure .6shows the 3D FEM analysis results under design load, with relative displacements taken into account. As shown in the case without filling (left) and the case with filling (right), minor principal stress at the tips of the ribs was improved by filling with mortar from –95.4 N/mm^2 to –65.1 N/mm^2, or to 68%, indicating successful dispersion of stress.

Although stress amplitude under live load was only 2.0% of that under full design load by the overall frame analysis, full penetration weld joints were adopted for the steel pipes for assured safety against fatigue in the struts.

Safety against buckling in the struts was verified by a buckling ratio of 13.5 in the primary buckling mode.

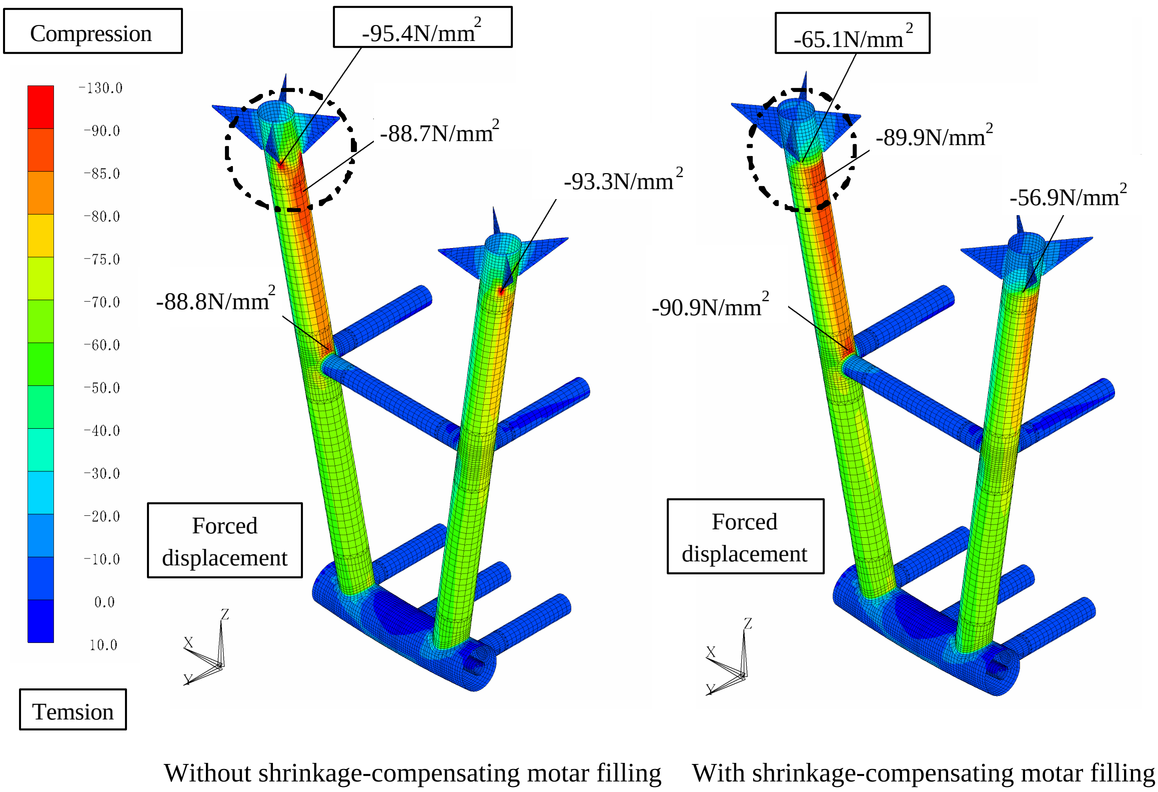

Fig. 6 3D FEM analysis on the struts

(2) Horizontal rubber bearings in the central hinge region, and reinforcement of the existing cross beams

The hinge in the existing bridge consisted of Gelenk shoes which transferred shear force only and did not transfer bending or axial force. When the current reinforcement was simulated without modifying the existing hinge, the bottoms of the existing piers could not bear the specified stress due to the prestressing and temperature change. In order to allow axial force to be transferred to the central hinge region, horizontal rubber bearings were installed. Steel shoe seats were used for ease of work.

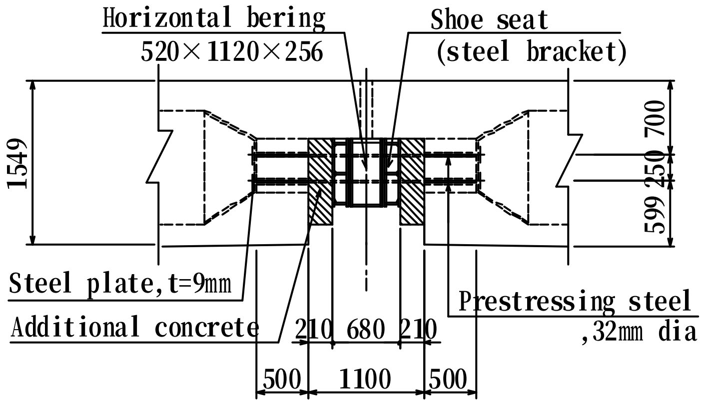

Fig. 7 Details of the central hinge region (side view)

Axial force acting on these horizontal bearings was about 6350 kN/shoe under design load (temperature change and variation), suggesting that the existing cross beams were insufficient to obtain adequate flexural and punching shear capacities. Therefore, they were reinforced with additional concrete and bonded steel plates as shown in Fig. 7.

(3) King post cable anchorage zones

The king post cables were anchored to the existing pier head regions. Anchors and deviators were installed by placing additional concrete. The thickness of the additional concrete was set to 0.75 m for the front of the anchorage zones due to the layout of anchor components and, across the pier head region, 0.6 m for the deviation zones in order to maintain cable angle settings.

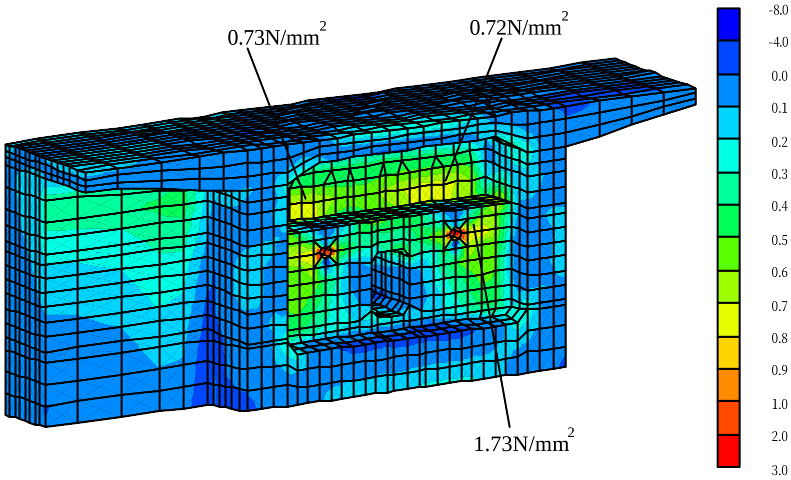

Fig. 8 3D FEM analysis results on the king post cable deviation zone

Local stress of this region was investigated by the 3D FEM analysis. Major principal stress was σ_{max} = 1.73 N/mm^2 in the vicinity of the anchors, exhibiting no deleterious tensile stress. Reinforcing bars were added to cope with the tension. Figure. 8 shows the 3D FEM analysis results on the king post cable deviation zones.

(4) Natural vibration characteristics

In an externally prestressed structure, structural vibration due to travelling vehicles and other factors is amplified by the resonance with external cables when the frequency of the free length of the cables is close to the lowest order deflection frequency of the main girder. Therefore, in order to confirm vibration characteristics after the reinforcement and the validity of the vibration controller spacings, natural vibration was analyzed using a plane frame model, taking shear stiffness due to the tension in the king post cables into account. As shown in Fig. 9, the frequency of the external cables (f_c) was larger than five times the frequency of the main girder (f_h) ($5f_h < f_c$), suggesting that no deleterious vibration accompanied by resonance would occur[6]. It was decided to install vibration controllers at spacings of less than 10 [7] to increase safety against resonance which was already assured by the analysis results even without them.

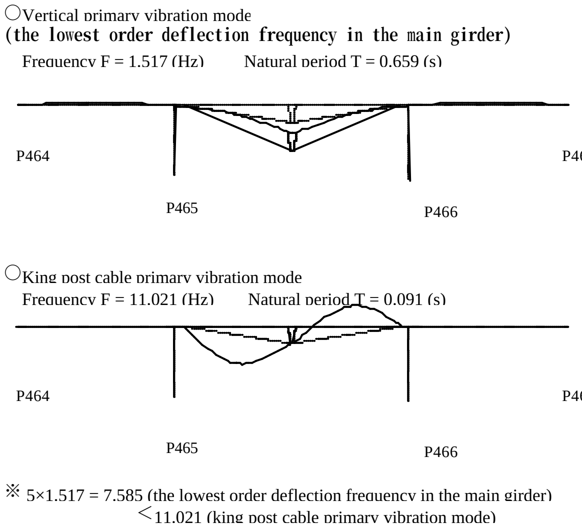

Fig. 9 Natural vibration analysis results

4. Conclusion

Reinforcement using king post cables was examined to prevent or correct sag in the central hinge region of a rigid frame box girder. There were following findings through its detailed design.

(1) Updating of the existing bridge design with current standards: When B live load was applied, tensile stress under minimum design load exceeded a specified level partly on the central span top fiber before the reinforcement but was controlled within a satisfactory range in every cross-section after the reinforcement.

(2) Combined stress: Every cross-section was fully prestressed after the reinforcement, with a minimum compressive stress in the bottom fiber around the central hinge region and a margin of 2.0 N/mm^2 from an allowable level in tensile stress. The amount of recovery from sag was 47 mm at the central hinge region, suggesting that proper layout and tension were selected for the king post cables.

(3) Design of the struts: Stress concentration was reduced analytically to about 68% by filling steel pipes at their roots with shrinkage-compensating mortar. This point will be confirmed during the measurements on the existing bridge.

(4) Relative displacement across the central hinge: There was a vertical displacement of 0.96 mm between the two sides. Although safety was confirmed during the designing of the struts with a forced displacement of 5 mm, future maintenance should include inspection of the Gelenk shoes which transfer shear force in order to evaluate their wear over time.

(5) The additional concrete reinforcement on the cross beams in the central hinge region which was required for installing the horizontal rubber bearings resulted in a slight increase in dead load but also provided an intermediate for a plane contact between a steel shoe seat and the existing cross beam, eliminating the concern over the installation accuracy of the horizontal rubber bearings..

(6) Natural vibration characteristics: Safety against resonance was assured after the reinforcement even without vibration controllers.

5. Future examinations

5.1 Design verification based on the property investigation results

The current detailed design is a basis established based on the property values from previous investigations. Further investigations are under way at present for collecting property data, and measurements are planned to obtain residual prestress data which are not sufficient at this moment. Design verification will be carried out using these additional data which will represent the condition of the existing bridge more accurately.

5.2 Measurement plans for the existing bridge

Previous data was insufficient to identify the major factors of the sag occurring in the central hinge region. Since the current reinforcement technique has not been proven by actual applications, it is necessary to confirm if the existing bridge behaves as calculated in the design. Following measurements are planned for both design verification and future maintenance.

(1) Displacement in major members and stress in the main girder cross-section will be measured during prestressing so that tension in the king post cables will be controlled at real time.
(2) Static loading tests will be performed before and after prestressing in order to verify effect of the current reinforcement and behavior caused by live load.
(3) Follow-up investigations will be carried out to confirm preventive effect against sagging and establish proper maintenance plans, so that soundness after the reinforcement will be evaluated and basic data required for preparing maintenance manuals will be collected.

Reference

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